Lifespan evaluation of 8 bridges of the Indiana Toll Road. A case study

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ABSTRACT: This paper shows the practical experience achieved by evaluating the lifespan of some existing structures under extreme weather conditions, in order to decide on either demolition or repair. The applied methodology shows the feasibility of the approach based on "durability engineering" according to mainframe of current models of degradation. Thus, by using both relatively conventional testing methods and the mentioned available models, it was possible to estimate the remaining lifetime of these structures. Nevertheless, some difficulties appear when applying theory to practice, as pointed out in this paper.

1 DESCRIPTION OF STRUCTURES

The Indiana Toll Road is a part of the New York - Chicago Toll Road system running east-west across the upper north side of Indiana. The eight analysed structures share similar typologies and are exposed to the same extreme weather and aggressive environment conditions. All eight structures were built in 1956 and underwent repair and widening operations during the 1980's. Table 1 contains the main geometrical dimensions of these structures.

Table 1. Summary of the structures analized.

Structure	Number of spans		Length (ft)	Width (ft)	Area (sf)	% Length over total	
I-12, R.R. tracks and Gary W.T.P.	WB	40	3,013.70	35	105,480	18.21	36.4
	EB		3,013.70	35	105,480	18.21	
Grand Calumet River West	WB	3	216.43	35	7,575	1.31	2.62
	EB		216.43	35	7,575	1.31	
Bridge Street	WB	7	340.85	35	11,930	2.06	5.36
	EB	11	546	35	19,110	3.3	
Grand Calumet River East	WB	5	165.59	42	6,955	1	2.01
	EB		167.47	43.3	7,251	1.01	
Buchanan Street	WB	3	218.74	42	9,187	1.32	2.64
	EB		218.74	53.3	11,659	1.32	
Broadway St., Virginia St., E.J. & E. R.R.	WB	75	3,941.29	41.2	162,381	23.82	47.6
	EB		3,941.29	41.2	162,381	23.82	
Tennessee Street	WB	3	128.13	41.7	5,343	0.77	1.55
	EB		128.13	41.7	5,343	0.77	
Old Hobart Road	WB	3	146.49	35	5,127	0.89	1.77
	EB		146.49	35	5,127	0.89	
			16,549.47		637,904	100	100

Their general typological properties may be summarized as follows:

Most of these structures are made up of twin decks of almost equal dimensions. Each structure consists of statically determined decks of a reinforced concrete slab on several longitudinal steel stringers, simply supported on abutments and bents. Every deck shows transversal joints on abutments and bents.

Bents are made up of square reinforced concrete piers and rectangular lintels, with the exception of the Broadway St structure, containing a combination of R.C. elements and steel struts. Bents are founded on reinforced concrete pile-caps.

Abutments are closed, with a breast wall founded on piles, transversal wing walls and a back slipper on piles supporting an approach slab.

Although the Indiana Toll Road suffered several repair works in the past, the effectiveness of the maintenance was rather poor.

2 GENERAL CONDITION OF BRIDGES

The structural steel has undergone different levels of degradation.

Concrete shows evidences of damages not induced by steel corrosion. The local freezing-thaw effects of de-icing salts produced successive scaling of concrete (figure 1, left). Water leakage with de-icing agents also favours the circulation of liquid water, even under low temperatures, with an important amount of chlorides, which ineluctably leaded to corrosion of reinforcement (figure 1, right).

It is worthwhile mentioning that damages on the original structures (1950's) and on the broadened ones (1980's) were very similar. This analogous condition is probably due to a similar time of exposure to de-icing agents, extensively applied after the second part of the 1960's.

Figure 1. Left: concrete scaling, threatening the stability of a bearing device. Right: chloride induced corrosion attack.

It is also interesting to observe that past concrete repairs usually show map cracking, probably due to either exothermic reactions (the thickness of the repaired zones is relatively high, about 10-15 cm) or to drying shrinkage due to poor curing. In any case, it is necessary to point out that the technical basis for durability knowledge and strategies of intervention are relatively new (about twenty years), which left former repair interventions without a proper diagnosis. In addition, although undoubtedly the main reason for corrosion on structural steel girders and concrete bents and piers lies on the poor ability of joints to keep water away, another reason to explain the apparent lack of efficiency of the repaired concrete cover is to be found on the formation of macro-cells between apparently healthy zones, formerly cathodic zones, and the repaired zones, formerly anodic and transformed into well protected cathodics.

No significant structural damages (differential settlements, shear cracks, etc.) were detected, highlighting the fact, once again, that the vast majority of structures arrive on the end of their lifetime after durability problems. In some cases, vertical cracking due to horizontal restrained shrinkage was found at abutment walls. Although the crack width is about 0.3 mm, no special evidence of corrosion of reinforcement is associated to such cracking.

Transverse cracking at top slabs (perpendicular to main girders) is due to the lack of sufficient longitudinal reinforcement to control crack width. Anyhow, cracks are small and do not necessarily coincide with corroded areas.

Another interesting case is the special structure over Broadway St. Petrographic analysis detected the existence of chert, sensitive to Alkali-Silica-Reaction (ASR). In fact, it seems that gel is already appearing and filling up the neighbouring voids of these fine aggregates, but the potential effects on these structures' future service life are negligible in practice.

Obviously, the intensity and extent of the damages are closely related to the state of joints and the drainage system.

3 AUSCULTATION PLAN AND RESULTS

3.1 *Auscultation plan and taking of cores*

Several techniques were used in order to measure the significant properties related to structural and durable behaviour:

a) Tests to quantify both structural parameters and the extension of damages (acoustic sounding, impact echo and spectral analysis of surface wave and ground penetrating radar).
b) Tests to characterize mechanical properties of materials (compressive strength, Ultrasonic Pulse Velocity, Rebound Index and density for concrete and the tensile strength, resilience and weldability for structural steel).
c) Tests to identify properties related to the durability of materials (chloride contents profiles, carbonation depth, chemical water analysis of the Grand Calumet River in order to quantify its potential aggressivity, electrical half-cell potential of reinforcing steel, corrosion rate (I_{corr}) by Galvanostatic Pulse Method and estimation of concrete cover of reinforcement from the ground penetration radar readings.

Table 2. Summary of cores taken and related ratios.

Structure	Cores	Deck surface (m^2)	Ratio (cores / deck area m^2)
Gary	25	34,483	1/1,380
Grand Calumet West	10	2,476	1/250
Bridge Street	14	5,073	1/360
Buchanan Street	19	2,910	1/150
Grand Calumet East	14	1,376	1/100
Broadway	30	41,797	1/1,400
Tennesee Street	15	1,359	1/90
Old Hobart Road	11	1,819	1/170
Total identified cores	*138*	*91,320*	*1/660*

138 cores were taken: 57% from slabs and 43% from substructures (abutments and bents), as summarized in table 2.

3.2 *General criteria for auscultation results*

Since all eight structures possess similar typologies, were built during the same period of time and, apparently, with the same materials, as well as their exposure to analogous environmental factors, a global layout for all structures may be considered valid for all of them. Thus, general strategies are valid for all cases and to be applied to each type of damage in any of these structures. Unfortunately, obtained results do not confirm this hypothesis, as will be explained in the following paragraphs. Therefore, it was necessary to define a specific strategy for each and every structure.

Firstly, an analysis of the mechanical properties of concrete was carried out in order to identify the types of concrete used for each structure. From the 'as built' project could be deduced that class D concrete was done using 1.50 barrels of cement per cubic yard, while 1.25 and 1.75 barrels where used for concrete classes E and F, respectively (a barrel equals to 375 lb of cement).

Afterwards, an analysis of densities was made. According to design two types of concrete can be clearly distinguished: class F used for decks and class D for substructures. The specified concrete strength was 4,000 psi (27,6 MPa) for decks and 3,500 psi (24,1 MPa) for substructures. Figure 2 summarizes results.

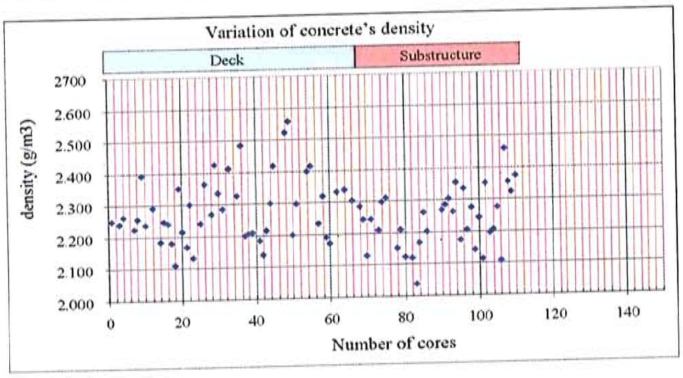

Figure 2. Variation of concrete densities. Abscissas: sequence of cores. Ordinates: obtained density of each core.

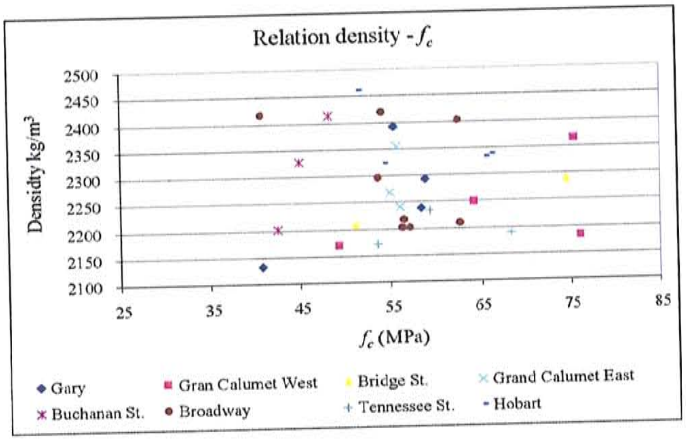

Figure 3. Scatter of concrete densities and strength.

Despite the scatter, a great heterogeneity of values was also detected, with some apparent inconsistencies, particularly after analysing the relationship concrete strength vs density, as shown in figure 3.

In relation to the evolution of concrete strengths of decks and substructures, the following expression relates design vs obtained mean, real and characteristics values.

$$\frac{f_{ck,deck,real}}{f_{ck,substructure,real}} = \frac{59.08-1.645\times 8.88}{49.01-1.645\times 6.03} = 1.138 \approx \frac{f_{c,deck,project}}{f_{c,substructure,project}} = \frac{4000psi}{3500psi} = 1.143$$

$$\frac{\overline{f}_{c,deck,real}}{f_{c,substructure,real}} = \frac{59.08}{49.01} = 1.21 > \frac{f_{c,deck,project}}{f_{c,substructure,project}} = \frac{4000psi}{3500psi} = 1.143$$

It seemed that both types of concrete equally developed their characteristic strengths. In order to explain the obtained scatter, it was assumed that one part of the concrete might have been produced by using different air-entraining admixtures. To confirm it, the decrease of strength was analysed as a function of the decrease of density by accepting the hypothesis of different percentages of air-entraining admixtures (figure 4). After literature, 1% of air-entrained represents a decreasing strength up to 4% and 5%.

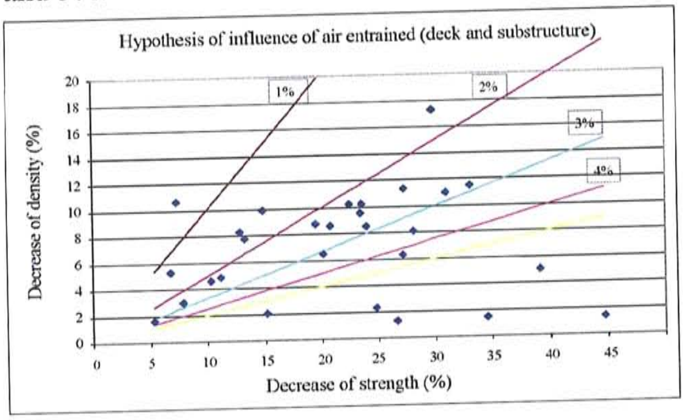

Figure 4. Potential influence of the air-entraining percentage on decreased strength.

As a consequence, the hypothesis of a deck made up by air-entrained admixtures and a substructure free of them seems uncertain. In addition, it seems that any degradation process acting along the life of the structure may have affected both decks and substructures in a homogeneous way (appearance of freezing-thaw processes, etc.). Summarizing, the obtained results denote great heterogeneities, much greater than expected from similar structures with analogous concrete specifications in project and built during the same period. As a consequence, the initial methodology of adopting standard solutions and the hypothesis of similar behaviour among the different structures could not be maintained. Thus, a specific study for each structure had to be carried out.

4 GENERAL CRITERIA ADOPTED

4.1 *Exposure conditions*

All analyzed structures are located in the same geographical area, which is exposed to a rather extreme climate, with extremely low temperatures during several months in late autumn and winter. According to its annual temperatures and rainfalls the massive use of de-icing agents to maintain traffic is evident, which may be responsible for the development of chloride induced corrosion mechanisms on reinforcing bars and specific concrete degradation.

4.2 *General criteria for lifespan estimation*

In this document, the adopted criteria to estimate the lifespan of these structures are based on the fact that the conditioning phenomenon for their degradation process is chloride induced corrosion of reinforcing bars, since this type of attack is predominant and is, therefore, strongly conditioning their life expectancy.

Accordingly, the methodology to estimate the lifespan of the structures follows these steps:

a) Analysis of experimental results, in order to identify possible uncertainties or mistakes.
b) Modelling of the main cause for degradation (corrosion induced by de-icing agents) and estimation of the relevant parameters for material behaviour (chloride diffusion coefficient).
c) Estimation of limit conditions of durability of the structures under real exposure conditions (chloride threshold or limit value of chloride contents leading to corrosion under real moisture and temperature conditions).
d) Estimation of the time-dependent model of behaviour to describe relevant mechanisms of durability (estimation of chlorides distribution in time and depth).
e) Estimation of the remaining lifespan, that is, the age at which the durability conditions can no longer be fulfilled: elapsing time when chloride contents at reinforcement level reach the aforementioned threshold value for real environmental conditions).

Therefore, in order to minimize possible uncertainties of this layout, it is essential to count maximum warrants of the carried out experimental determinations for the true identification of chloride contents, cover values and half-cell potentials. In the particular case of diffusion coefficient estimation of the, accuracy depends on sufficient number of valid data from tests, that is, sufficient measures of chloride contents per core.

4.3 *Modelling of the chloride induced degradation process*

The accepted model for the diffusion of chlorides across the concrete mass, under non-stationary conditions, is governed by Fick's second law:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}\left(D\frac{\partial C}{\partial x}\right) \tag{1}$$

This equation expresses the flux of concentration C of the diffusing species (chlorides) at a depth x from the outer surface along time t. D is the diffusion coefficient, expressed in m^2/s, which depends on concrete characteristics and on environmental conditions.

An approximate solution is given by

$$C_x - C_b = (C_s - C_b)\left[1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)\right] \tag{2}$$

with the following meanings:

C_s contents of chlorides Cl^- at convection zone;
C_x contents of chlorides Cl^- at a distance x from the outer concrete surface; and
C_b contents of chlorides Cl^- already present in concrete mass and provided by its components;

In order to apply this model, it is necessary to obtain the diffusion coefficient D (which may typically vary from 10^{-11} and 10^{-13} m^2/s as a function of concrete characteristics). As stated above, the precision of such sensitive value depends on available information. Therefore, when the number of chloride contents per concrete core at a given position is limited to 3 or 4, the precision may be reduced, especially considering that a convection zone is to be expected due to variable environmental conditions. In addition, it is essential to correct the chloride contents in order to consider their already present amount at construction, as illustrated in figure 5.

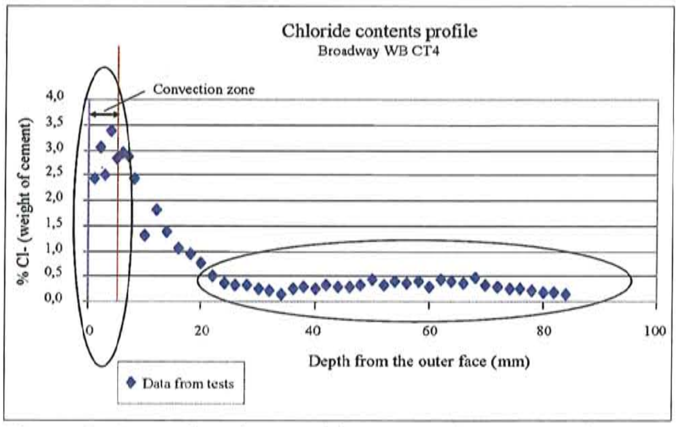

Figure 5. Corrections imposed by convection zone and components effect.

Therefore, for any determination on the different test cores, it is necessary to identify the region that really provides the diffusion of chlorides, disregarding the results outside This procedure implies the movement of both axes in figure 5. The distances are C_b for y-axis and x_{conv} for x-axis, being x_{conv} the depth affected by the above described convection mechanism. The obtained chloride profile is analysed by optimising the fit of the approximate equation given above by using the least square method after a linearization of the equation throughout a variable transformation (figure 6).

Indeed, the diffusion coefficient is not constant and may change in function of position and time, following variations in the pore structure or due to external humidity and temperature. This variation in D may be modelled by means of an ageing function given by:

$$D(t) = D(t_0)\, A(t) \quad \therefore \quad A(t) = \left(\frac{t_0}{t}\right)^m \tag{3}$$

In the case of the structures analysed in this document, t_0 = 50 years (current age after construction); and t = 100 years (50 years of remaining desired life). Exponent m depends on type of cement. According to indications of *fib*, an m value of 0.50 has been adopted.

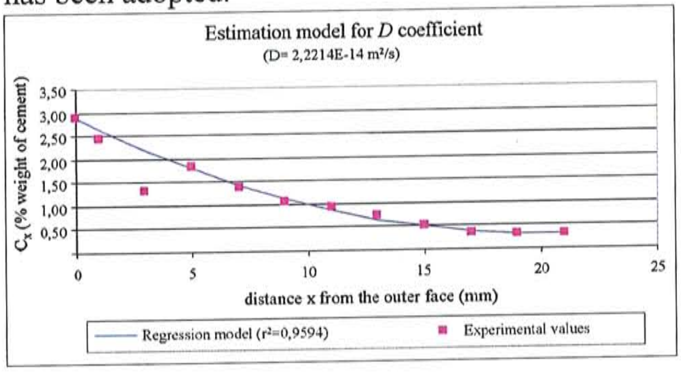

Figure 6. Example of regression in one of the cores.

The chloride threshold for initiation of pitting corrosion for a given structure depends on numerous factors, i.e. pH of concrete, electrochemical potential of steel and presence of voids at steel/concrete interface. Traditionally, the threshold chloride content that may produce electrochemical reactions of corrosion corresponds to the weight of cement and is related to the binding capacity of the cement paste, among others. Therefore, according to *fib* recommendations, an average value of 0.6% (respect to mass of cement) for such a threshold is adopted. However, it is frequent to find some discrepancies or anomalous results provided by the labs, as happened in this case. This problem could be eluded since some measures, such as the corrosion potential, were also available. This means that if cement content values could not be determined, chloride contents should be related to concrete weight instead of cement weight.

As an important number of half-cell potential, basic parameter for the quantification of electrochemical phenomenon, were measured, an alternative procedure was adopted to quantify the chloride threshold throughout the relationship between half-cell potentials, chloride contents and the existence of damages at the positions where such measurements were taken. It therefore seems that the presence of damages could be related to threshold values corresponding to half-cell potential of -300 mV and 0.085% of chloride content of the mass of concrete. It is worth mentioning that this percentage perfectly fits the theoretical value of 0.6% recommended by *fib*, although expressed in terms of chlorides mass with respect to mass of cement in the case of a cement contents of about 333 kg/m^3, which is the value defined in the project of substructure elements, where the presence of damages is greater.

4.4 *Estimation of time-dependent evolution of chloride penetration*

The model of Tuutti (1982) is accepted as a basis of this strategy. Usually, propagation time is negligible in relation to the initiation period, so t_1 = 0 has been adopted.

The initiation period t_0 is conditioned by the chloride diffusion processes from the outer surface to the reinforcing steel bars. From equation (2) derives:

$$t_0 = \frac{1}{12D}\left[\frac{r_{min}}{1-\sqrt{\dfrac{C_{lim}-C_b}{C_s-C_b}}}\right]^2 \tag{4}$$

Thus, for each structure it is necessary to give diffusion coefficient D, concrete cover 2” (corrected in order to consider the convection zone (0,5”) due to the fact that the maximum chloride content does not correspond to the surface but to a depth of 0,5”, that is, r_{min} = 2 – 0,5 = 1,5”, a chloride threshold value C_{lim} = 0.085% (referred to the mass of concrete, equivalent to 0.6% in mass of cement), and a chloride concentration C_s measured at the surface (maximum value, for a already elapsed time of 50 years, equal to the desired remaining lifespan) equivalent to 0.31% referred to the mass of concrete. Chloride concentrations C_b provided at construction stage by the cement and aggregates correspond to the horizontal line measured at reasonable depth, in order to ensure their independence from chloride diffusion mechanisms.

To assess the lifespan with a given failure probability, it is possible to develop a ‘semi probabilistic’ method, similar to the usual limit states method for structural assessment. For the purpose of this study, a probability failure p_f = 0.50 has been considered. A partial safety factor for lifespan γ_L=1.1 was adopted, according to the new Spanish Code.

5 GENERAL CRITERIA TO DECIDE ON REPAIR STRATEGIES

The election of an adequate repair strategy had to be based on the data collected during the auscultation phase, and was based on the following methodology. Figure 7 shows the chart of decisions and explains the significance of all related proposals of intervention. The process starts with the intensity I_{corr} measurement of the electrical current on bars.

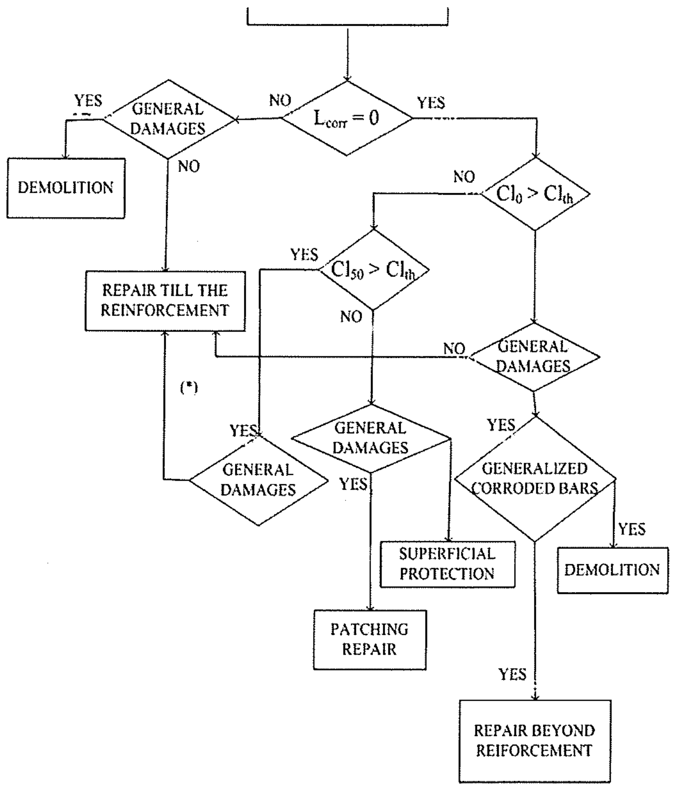

Figure 7. Flow-chart to decide the repair strategy.

If $I_{corr} = 0$, no corrosion process is taking place at the moment. Thus, it is necessary to compare chloride contents at level of reinforcement Cl_0 with the chloride threshold Cl_{th}. If $Cl_0 > Cl_{th}$, showing no evidence of generalized damages, delamination of concrete and acceleration of corrosion will be taking place shortly after. In that case, the only possible intervention is to eliminate the contaminated concrete cover even behind reinforcement (according to the distribution of chlorides), although there are also different strategies available, as i.e. cathodic protection. If $Cl_0 > Cl_{th}$, the concrete member exhibits generalized damages and its reinforcement shows generalized corrosion too, with a significant loss of section, demolition would be the best option. If $Cl_0 > Cl_{th}$ and the concrete member shows generalized damages and its reinforcement shows a fine layer of rust, but without relevant loss of section, the elimination of the deteriorated concrete even behind the reinforcement shall be necessary.

If Cl_{50} is the foreseen chloride contents at the level of reinforcement (according to available models, as explained above) at time t=50 years. If $Cl_0 < Cl_{th}$, $Cl_{50} > Cl_{th}$ or $Cl_0 < Cl_{th}$, $Cl_{50} < Cl_{th}$ and the concrete member exhibits generalized damages (i.e. due to freeze-thawing effects), the removal of the concrete cover and its repair with new concrete is proposed. If $Cl_0 < Cl_{th}$, $Cl_{50} > Cl_{th}$ and the concrete member does not shows generalized damages, patching repair may be carried out.

If $Cl_0 < Cl_{th}$, $Cl_{50} > Cl_{th}$ and the concrete member does not exhibit generalized damages painting with corrosion inhibitors or watertight layer may be applied to the existing surface.

If $I_{corr} > 0$, a corrosion process is taking place. If, in addition, the concrete member shows generalized damages, demolition might be inevitable. On the contrary, if no generalized damages are present, the above mentioned technique of removal the old cover and its replacement by a new one is proposed.

6 CONCLUSIONS

The current knowledge of the durability models for degradation processes allowed the development of effective strategies for the assessment of the remaining lifespan in existent structures. The applied methodology for the Indiana Toll Road bridges shows the feasibility of this approach based on "durability engineering". By using relative conventional testing methods, it was possible to assess the remaining lifetime for the different structures located in very aggressive environment conditions. Nonetheless, some practical difficulties appeared when auscultation methods were carried out and theoretical models were applied. The main identified problems were the following:

- The number of cores must be limited, although representative in order to minimize the scattering of results, claiming to coordinate both auscultation and engineering activities.
- On some occasions, the obtained results did not fit expected theoretical behaviour. In these cases, the application of several simultaneous techniques for significant parameters may be highly effective.
- In practice, the application of Fick´s second law is associated to the assumption of values for some parameters which are very sensitive. Chloride threshold for corrosion and surface chloride contents may be good examples of that.
- The decision making process from obtained lifespan values may be defined according to an adequate repair strategy.

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